# CHAPTER 4. STRESS ANALYSIS

## Section 4-1 Introduction

### 4-1.01 Policy Statement

The design stresses and deflections set forth in the falsework specifications are the maximumstresses and deflections which may be allowed for a given loading condition. Loads are to be applied in accordance with the policy and procedures discussed in Chapter 3, Section 3-l. Individual members of the falsework system, as well as the system as a whole, must be capable of resisting the specified design loads without exceeding the allowable values.

In accordance with Division of Structures policy, falsework drawings are not to be approved in any case where the calculated stress or deflection in any falsework member exceeds the allowable stress or deflection for that falsework member.

### 4-l.02 General Design Assumptions

In general, stresses in load-carrying members of the falsework system may be determined by using the general formulas of civil engineering design applicable to statically determinate framed structures.

For those elements of the falsework system that are statically indeterminate, the Division of Structures has developed specific methods and procedures which are to be used when investigating system adequacy. These procedures, which are applicable to diagonal bracing, metal shoring systems, and pads and pile bents, are explained in Chapters 5, 6 and 7, respectively, of this manual.

The load-carrying capacity of commercial products or devices, such as jacks, beam hangers, deck overhang brackets and similar items, should be determined reference to a catalog, brochure or other technical literature published by the manufacturer, or by a load test performed in accordance with the instructions in Section 4-6, Manufactured Assemblies.

The load imposed on falsework beams and stringers by the slab support system of closely-spaced joists is actually applied as a series of concentrated loads. When calculating stresses in these members, an equivalent uniform load may be assumed.

The effect of beam continuity must be investigated. As provided by Division policy, any theoretical advantage resulting from continuity should be neglected; however, the adverse effects must be considered to prevent overstressing of any falsework member, (See discussion in Chapter 3.)

When calculating stresses, keep in mind that the individual falsework members as well as the system as a whole must be capable of resisting all imposed loads, including any direct or redistributed load caused by beam continuity, the construction sequence, prestressing, deck shrinkage, and similar design and construction features which may contribute to the overall load to be carried by the member under investigation.

## Section 4-2 Timber Members

### 4-2.01 Member Size

Timber members should be assumed as S4S unless shown otherwise on the falsework drawings.

The dimensions of rough cut lumber may vary appreciably from the theoretical dimension, particularly in the larger sizes commonly used in falsework construction. If the use of rough cut lumber is anticipated by the falsework design, the actual member size must be verified prior to use.

### 4-2.02 Allowable Stresses and Load Duration

The maximum allowable stresses for timber as listed in the specifications include an adjustment for an assumed duration of load of about ten days, which is typical for most falsework installations. Since these stresses are maximums, they may not be increased even though the actual duration of load may turn out to be less than the assumed duration.

Occasionally a situation will occur where the falsework will be loaded for a long period of time, such as, for example, when a continuous structure is constructed in stages. In such cases load duration considerations may warrant a reduction (by the contractor) in the allowable stress level.

Appropriate duration of load factors are shown in Figure 4-6.

### 4-2.03 Timber Beams

#### 4-2.03A Beam Span

For simple beams the span length is the clear distance from face-to-face of supports, plus one-half the required bearing length at each end.

For continuous beams the span length is the center-to-center distance between supports over which the beam is continuous. For end spans of-continuous beams the span length is the distance between the center-of-bearing at the continuous support and the point of end support determined in accordance with the simple beam rule stated in the preceding paragraph.

4-2.03B Bending and Deflection

The extreme fiber stress due to bending ($f_b$) is calculated from the formula:

$$f_b = Mc/I \quad \text{or} \quad f_b = M/S$$

where $f_b$ is the bending stress in pounds per square inch; M is the bending moment, in inch-pounds; c is the distance from the neutral axis to the extreme fiber, in inches; I is the moment of inertia of the section about the neutral axis, in inches $^4$; and S is the section modulus, in inches$^3$.

Deep, narrow beams may require lateral support to prevent the compression edge from buckling before the allowable bending stress is reached. (See Section 4-2.03E, Lateral Support of Wood Beams.)

The maximum deflection ($\Delta$) of a uniformly loaded simple beam is given by the formula:

$$\Delta = \frac{5WL^3}{384\ EI}$$

where $\Delta$ is the deflection, in inches; W is the total uniformly distributed load, in pounds; L is the beam span, in inches; and E and I are, respectively, the modulus of elasticity and the moment of inertia, both in customary units.

4-2.03C Horizontal Shear

The formula for horizontal shear in a rectangular beam "b" inches wide and "d" inches deep is:

$$f_v = 3V/2bd \quad \text{or} \quad f_v = 3V/2A$$

where $f_v$ is the maximum horizontal shearing-stress, in pounds per square inch; V is the vertical shear, in pounds; and A is the cross sectional area of the beam, in square inches.

Theoretically, the strength of a wood beam in horizontal shear is a function of that strength property for the specie and the extent to which a particular beam may be checked or split at the end, However, tests by the U.S. Forest Products Laboratory and others have shown that with split beams the shear force is not uniformly distributed as assumed by the shear formula. Instead, in a split or checked beam, the upper and lower halves of the beam each resist a portion of the total horizontal shear force independently of the force resisted by the beam at the neutral axis, so that a split or checked beam is capable of carrying a larger load than would appear to be the case using the general shear formula. Investigation of this phenomenon led to the derivation of so-called "two-beam" or "checked-beam" formulas from which the horizontal shearing stress may be determined with greater accuracy.

When reviewing falsework designs, the horizontal shearing stress should be computed using the general formula for a rectangular section [1]. When computing the total shear "V" to use in the formula, neglect all loads within a distance from the face of the support equal to the depth of the beam. If the allowable stress is exceeded when computed by the general formula, and if the contractor's beam design is based on the use of the checked-beam method of analysis, the shear value "V" may be determined by using the checked-beam formulas, and this value used in the horizontal shear calculation [2].

4-2.03D Compression Perpendicular to the Grain

Compression perpendicular to the grain at beam supports is given by the formula:

$$f_{c\perp} = P/A$$

where $f_{c\perp}$ is the compression stress perpendicular to the grain, in pounds per square inch; P is the applied load, in pounds; and A is the-bearing area in square inches.

When calculating the bearing area at the end of a beam, no allowance need be made for the fact that, as the beam deflects, the pressure at the heaviest loaded edge of the support appears greater than at the other edge. The wood yields enough so that pressures equalize, and overstressing does not occur.

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[1] Note that the Division's Falsework Check computer program uses the general formula for rectangular sections to calculate horizontal shear.

[2] A discussion of checked-beam theory is not included in this manual because horizontal shear is seldom critical in bridge falsework spans. However, a discussion of the checked-beam method of analysis may be found in the *National Design Specification for Wood Construction* and other timber design manuals, and reference is made thereto.

For small members (2x4's, 2x6's, 4x4's etc.) having a short length of bearing, it is standard practice to compute the bearing stress on the basis of an effective area determined by multiplying the actual area by the factor:

$$\frac{L + 3/8}{L}$$

where L is the bearing length in inches.

Use of the "effective area" factor will be permitted in the analysis of bridge falsework, provided the bearing length is less than six inches and the end of the load-carrying member extends three inches or more beyond the back face of the support.

To facilitate construction at locations where a conventional support may not be feasible, falsework members occasionally are supported by rods or dowels cast into a previous concrete pour. For example, lost deck forms may be supported- by a ledger beam bearing on dowels cast into the girder stem. In this or any other case where a timber member bears directly on a round support, there will be some yielding of the wood fibers as the load is applied, and some crushing will occur.

When investigating bearing adequacy. when timber members are supported by steel bars, the following policy will apply:

- When lost deck forms are supported by 2-inch nominal and wider ledger beams bearing on either 5/8-inch or 3/4-inch diameter reinforcing bar dowels, and provided the dowel extends far enough from the face of the concrete to-ensure full-width bearing under the ledger, a vertical design load of 900 pounds (maximum) may be used on each dowel.

- For all other cases where timber members bear directly on steel bars, bearing adequacy will be verified by means of an "equivalent length of bearing" equal to one-half the bar diameter. If the calculated stress based on an equivalent bearing length of one-half the bar diameter does not exceed the allowable stress, the detail may be approved even though some crushing will occur.

When investigating the bearing adequacy of a timber member on a round support, the bearing area obtained by using an equivalent length of bearing may not be increased further by applying the effective area factor previously discussed. Combining the two procedures would take unreasonable advantage of the bridging ability of wood fibers, and thus as a matter of policy will not be permitted for falsework analysis.

### 4-2.03E Lateral Support of Wood Beams

Beams having a large depth-to-width ratio may fail by lateral buckling (in much the same manner as long columns) before the allowable bending stress is reached unless they are restrained and forced to deflect in the plane of the load. The amount of restraint needed to ensure beam stability is a function of the depth-to-width ratio; however, it is not subject to precise analysis.

In the timber industry it is accepted design practice to check beam stability using guidelines developed empirically by the U.S. Forest Products Laboratory. However, these industry guidelines, which may be found in many timber design manuals and handbooks, were developed for permanent work and thus are too conservative for the temporary loading conditions typical of bridge falsework. Accordingly, the Division of Structures has modified the industry guidelines to reflect the temporary nature of falsework construction. The Division's criteria for evaluating timber beam stability are as follows:

- If the nominal depth-to-width ratio of a beam is 3:1 or less, no lateral support is needed.
- If the nominal depth-to-width ratio exceeds 3:l but is not more than 4:1, the ends of the beam should be braced at the top and bottom.
- If the nominal depth-to-span ratio exceeds 4:l but is not more than 6:1, the ends of the beam should be fully supported by blocking between beams.
- If the nominal ratio exceeds 6:1, in addition to blocking at the beam ends, blocking or diagonal bridging should be used at midspan for spans up to 16 feet, and at midspan and the quarter points for spans greater than 16 feet.

### 4-2.03F Beam Rollover [3]

When placed in other than a true vertical position, a timber beam will exhibit a tendency to rotate about its base as the load is applied. This rotational tendency, commonly referred to as "beam rollover", is an indication of instability which must be investigated during the falsework design review.

The tendency of beams to roll over when placed on a sloping surface is a function of the height and width of the beam,

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[3] As used in this section, the term "beam" means and includes any horizontal load-carrying member of the falsework system, including joists.

the load, and the slope (the angle with horizontal expressed as a percentage) on which the beam is placed. Beam rollover should be investigated in all cases where beams are set on a sloping surface using the procedure described below to find the maximum beam height for a given load, slope and beam width.

In addition to rollover stability, beams placed on a sloping surface require a further check to verify that the allowable compressive stress is not exceeded at the downslope corner of the beam.

4-2.03F(1) Investigation of Rollover Stability

For analysis of beam rollover, it is assumed that the vertical load acts as a concentrated load on the top center of the beam. Refer to Figure 4-l and note that the load transfers through the beam to the surface in contact with the supporting member through a vertical line. The beam will be stable against rollover if the line of the vertical load reaction lies within the beam width.

When moments are taken about the downslope corner of a beam placed on a sloping surface, as indicated by point A in Figure 4-1, the beam will be stable against rollover provided the righting moment (RM) exceeds the overturning moment (OTM).

For a given slope and beam load, there is a limiting beam height-to-width relationship. For a given width, the limiting height is determined as follows:

$$OTM = RM$$
$$h(P)\sin\phi = b/2(P)\cos\phi$$
$$(h)\tan\phi = b/2$$
$$h = b/2\tan\phi \approx b/[2(S/100)]$$
$$h \approx 50b/S$$

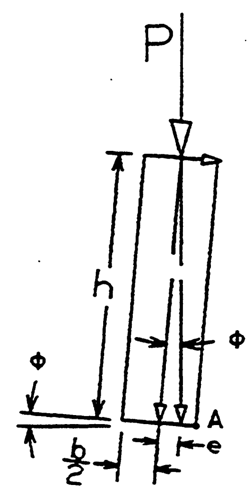


FIGURE 4-l

where P is the load on the beam, h is the height of the beam, b is the width of the beam, S is the slope expressed in percent, and $\phi$ is the tilt angle.

As an example, the limiting Slope for a 2x10 (1.5" x 9.25") beam is calculated as follows:

$$\text{Slope}(\%) = 50\frac{b}{h} = \frac{50(1.5)}{9.25} = 8.1\%$$

4-2.03F(2) Investigation of Compressive Stress

As the slope on which the beam rests increases, the compressive bearing stress between the beam and the supporting member at the downslope edge of the beam increases. This is the case because the center of gravity of the load acting through the top center of the beam remains vertical. The stress at the downslope edge is determined as follows:

- Calculate the compressive stress for normal bearing on the area between the beam and the supporting member as shown in Figure 4-2(a).

  $$f_c = P\cos\phi/A$$

  where $f_c$ is the compressive stress; P is the load and $\phi$ is the slope angle, so that $P\cos\phi$ is the load component acting perpendicular to the bearing surface; and A is the bearing area.

- Calculate the stress due to vertical load eccentricity using the bending stress equation. See Figure 4-2(b)

  $$f_e = Pe(\cos\phi)/S$$

  where $f_e$ is the stress due to the bending moment produced by the eccentric loading condition, e is the distance from the centerline of the beam at the bearing surface to the vertical reaction line, and S is the beam section modulus.

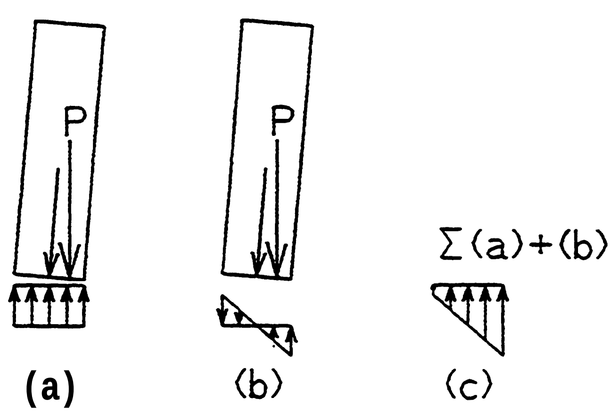


**FIGURE 4-2**

The sum of the stress values from steps 1 and 2 will give the total compressive stress at the downslope edge of the beam, as shown in Figure 4-2(c).

Except for joists bearing on camber strips, the calculated bearing stress on the downslope edge of a beam may not exceed the allowable stress of 450 psi. For joists bearing on camber strips, the calculated stress may not exceed the allowable stress of 900 psi.

As an example, the bearing stress on the downslope edge of a 2x10 (1.5" x 9.25") beam on a 6 percent cross slope resting-on a l-1/2" wide camber strip where the design load is 1900 pounds is calculated as follows:

$$\phi = \tan^{-1} 6/100 = 3.43°$$

$$e = h(\tan\phi) = 9.25(\tan 3.43°) = 0.555"$$

$$\text{Bearing Area} = 1.5(1.5) = 2.25 \text{ in}^2$$

$$\frac{P\cos\phi}{A} = \frac{1900 \cos 3.43°}{2.25} = 842.9 \text{ psi}$$

$$\frac{P\cos\phi ec}{I} = \frac{P\cos\phi e}{S} = \frac{1900 \cos 3.43°(0.555)}{\frac{bh^2}{6}} = 49.2 \text{ psi}$$

$$\text{Final Stresses} = 842.9 \pm 49.2 = 793.7 \text{ and } 892.1 < 900 \text{ psi}$$

## 4-2.03F(3) Blocking to Prevent Rollover[4]

Beams which otherwise would be unstable against rollover when investigated in accordance with the procedure described in the preceding sections may be made stable by the use of. full-depth blocking at the beam ends. Additionally, when the slope exceeds 8 percent, the following shall apply:

- If the nominal depth-to-width ratio is 4:l or less, blocking. should be provided at mid-span.
- If the nominal depth-to-span ratio exceeds 4:1, blocking should be provided at the l/3 points of the span.

For joists that are subject to rollover, toe-nailing to the supporting surface, in lieu of blocking, will not be permitted since the joist can break at the toe-nailed location.

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[4] The tendency of a beam to roll over is an independent condition of instability; consequently, the need for blocking to prevent beam rollover occurs independently of any requirement for blocking or other means of support to ensure beam stability as discussed in Section 4-2.03E, Lateral Support of Wood Beams.

### 4-2.04 Timber Posts

For analysis, falsework posts may be considered as pinned at the top and bottom, regardless of the actual end condition, except for driven pile bents. In a timber pile bent, the piles may be assumed as fixed at a point four diameters below the ground surface, unless the actual soil condition dictates a different assumption. (See Chapter 7 for the procedure to be followed when investigating the capacity of timber pile bents.)

For analysis, timber posts are classified as short, long or intermediate columns, depending on the failure mode.

Short columns are those in which failure occurs as a result of axial crushing, and bending is not a factor. Most authorities define a "short" column as one having a slenderness ratio of eleven or less. (Slenderness ratio is the ratio of length to least dimension.) Long columns are those in which failure is due to buckling, and strength is governed entirely by stiffness. Intermediate columns are those in which failure is due to a combination of axial crushing and bending.

In short posts the axial unit stress in compression parallel to the grain is determined by dividing the total load by the cross sectional area of the post; hence the formula:

$$f_c = P/A$$

where $f_c$ is the compressive stress parallel to the grain, in pounds per square inch; P is the axial load, in pounds; and A is the cross-sectional area of the post, in square inches.

Analysis of long and intermediate posts requires the use of empirical formulas to obtain a limiting value for $f_c$, which is less than the allowable for-axially-loaded short posts. These formulas are derived from the general Euler formula, which was developed for axially-loaded pin-ended columns. For a square or rectangular cross-section, the Euler formula is:

$$F_c = \frac{0.3E}{(L/d)^2}$$

where $F_c$ is the maximum allowable unit stress, in pounds per square inch; E is the modulus of elasticity; L is the unsupported length, in inches; and d is the minimum dimension, in inches, measured normal to the plane of bending.

The Euler formula reduces to the following when the modulus of elasticity for wood of 1.6 x $10^6$ psi is used:

$$F_c = \frac{480,000}{(L/d)^2}$$

The specifications limit the value of $F_c$ to 1600 psi. This limiting value corresponds to an L/d ratio of about 17.3.

In actual practice, the Euler formula gives conservative values since it is applied to restrained and square-end posts and columns, which are not as limber as the pin-ended columns from which it was derived.

4-2.04A Round and Tapered Posts and Piles

Other factors being equal, round and square posts having the same cross-sectional area have approximately equal stiffness, and therefore carry approximately the same load.

When analyzing round members, the least dimension "d" to use in the column formula should be taken as the length of the side of a square post whose area is equal to the cross-sectional area of the round post being used. This procedure will give results which are accurate within five percent for posts of the size and length ordinarily encountered in falsework construction.

When analyzing tapered members, the least dimension "d" to use in the slenderness ratio L/d is found by first determining the "equivalent diameter" of the tapered member by means of the following relationship:

$$D_0 = D_1 + \frac{D_2 - D_1}{3}$$

where $D_0$ is the equivalent diameter, $D_1$ is the tip or smallest diameter, and $D_2$ is the butt or largest diameter. All dimensions are in inches.

# Section 4-3 Timber Fasteners

## 4-3.01 Introduction

Design methodology and fastener values found in the *National Design Specification* for *Wood Construction* and other recognized timber handbooks are intended to apply to permanent work, and, thus are not necessarily appropriate for falsework. In view of this, the Division of Structures has developed alternative methodology that generally follows industry design practice, but which includes modifications where warranted to assure that the review procedures followed are reasonable in the light of false-work requirements.

## 4-3.02 Connector Design Values

Design values for both lateral load resistance and withdrawal resistance for wood fasteners in various wood species have been standardized by the timber industry, and are available in many timber design manuals and handbooks.

To facilitate review of falsework designs using timber bracing, fastener design values for Douglas Fir-Larch, which is the wood species commonly used for construction in California, are tabulated in Appendix E for nails, bolts, lag screws and drift pins. Design values for other fasteners, and for other wood species, may be obtained from the Sacramento Office of Structure Construction.

The design values in Appendix E are for normal duration of load, and may be increased for the shorter load durations typical of bridge falsework. See Section 4-3.07, Adjustment for Duration of Load.

## 4-3.03 Nails and Spikes

### 4-3.03A Design Values

Withdrawal and lateral load design values for nails and spikes are tabulated in Tables E-4 through E-7 in Appendix E.

The tabulated values *are* for an individual nail or spike. When more than one nail or spike is used in a connection, the total design value for the connection is the sum of the design values for the individual nails or spikes.

Diameters shown in the design tables apply to fasteners before application of any protective coating.

4-3.03(1) Withdrawal Resistance

The tabulated design values are for a nail or spike driven into the side grain of the member holding the point with the axis of the nail or spike perpendicular to the direction of the wood fibers.

Nails and spikes have little resistance to withdrawal when driven into the end grain of a wood member. Accordingly, the use of connections where the nail or spike is subject to withdrawal from the end grain of wood will not be permitted.

4-3.03A(2) Lateral Resistance

The tabulated design values are for a nail or spike driven into the side grain of the member holding the point with the axis of the nail or spike perpendicular to the direction of the wood fibers. The values apply for a lateral load acting in any direction.

When a nail or spike is driven into the end grain of a wood member, the design value is reduced to 2/3 of the tabulated value.

The ability of a nail or spike to resist lateral forces is a function of the diameter and depth of penetration of that nail or spike into the member holding the point. The design value tables show both the maximum and minimum lateral resistance values-for a given type and size fastener. Refer to the design tables (Tables E-4 through E-7 in Appendix E) and note that the maximum lateral resistance is reached when the penetration is 11 diameters, which is identified in the tables as the desired penetration. The lateral resistance value at the desired penetration of 11 diameters may not be increased, even though the actual penetration may exceed 11 diameters.

When the penetration is less than 11 diameters, the design value is obtained by straight-line interpolation between the maximum and minimum tabulated values, provided the actual penetration is not less than the minimum penetration shown.[5]

[5] A penetration of less than the desired penetration may occur when round posts are used or when longitudinal bracing on skewed bents is not parallel to the side of a square post. In such situations, care must be taken to ensure that the minimum penetration is actually obtained, since nails or spikes having a penetration of less than the minimum will have no allowable lateral load-carrying value.

4-3.03B Required Nail Spacing

The timber industry has not adopted standards to govern the spacing of nails and spikes when used in an engineered timber connection. The industry guideline is that edge distance, end distance and spacing must "...be sufficient to prevent splitting of the wood." *(National Design Specification for Wood Construction,* 1991 Edition.)

In recent years there has been a trend toward the use of a greater number of nails in timber falsework connections than would appear warranted by prudent design considerations. In view of this, the Division of Structures has established the following guidelines which, as a matter of policy, will govern the spacing of nails and spikes when used to connect falsework bracing components:

- The average center-to-center distance between adjacent nails or spikes, measured in any direction, shall not be less than the required penetration into the main member for the size of nail being used.

- The minimum end distance in the side member, and the minimum edge distance in both side member and main member, shall not be less than one-half of the required penetration.

While proper installation of timber connections is primarily a field concern, the design review must assure that the members are large enough to accommodate the required number of nails or spikes when the nails or spikes are spaced in conformance with Division of Structures criteria as set forth above.

4-3.03C Toe-Nailed Connections

Industry practice recommends that toe-nails be driven at an angle of approximately 30° with the member being toe-nailed, and started approximately l/3 the length of the nail from the end of the member.

Design values for withdrawal and lateral load resistance must be reduced for toe-nailed connections, as follows:

- For withdrawal loading, the design load shall be reduced to 2/3 of the value shown in the applicable design table.

- For lateral loading, the design load shall be reduced to 5/6 of the value shown in the applicable design table.

## 4-3.04 Bolted Connections

### 4-3.04A Design Values

Design values for bolted connections are shown in Table E-1 in Appendix E.

Except for connections in falsework adjacent to or over railroads, threaded rods and coil rods may be used in place of bolts of the same diameter with no reduction in the tabulated values.

The values in Table E-l may be used without modification when the load is applied either parallel or perpendicular to the direction of the wood grain. When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value for the main member is obtained from the Hankinson formula. The Hankinson formula is:

$$N = \frac{PQ}{P\sin^2\theta + Q\cos^2\theta}$$

where N is the design value for the main member; P and Q are, respectively, the tabulated design values for a load applied parallel to and perpendicular to the grain; and $\theta$ is the angle between the direction of the wood grain in the main member and the direction of the load in the side member. (See Figure 4-3.)

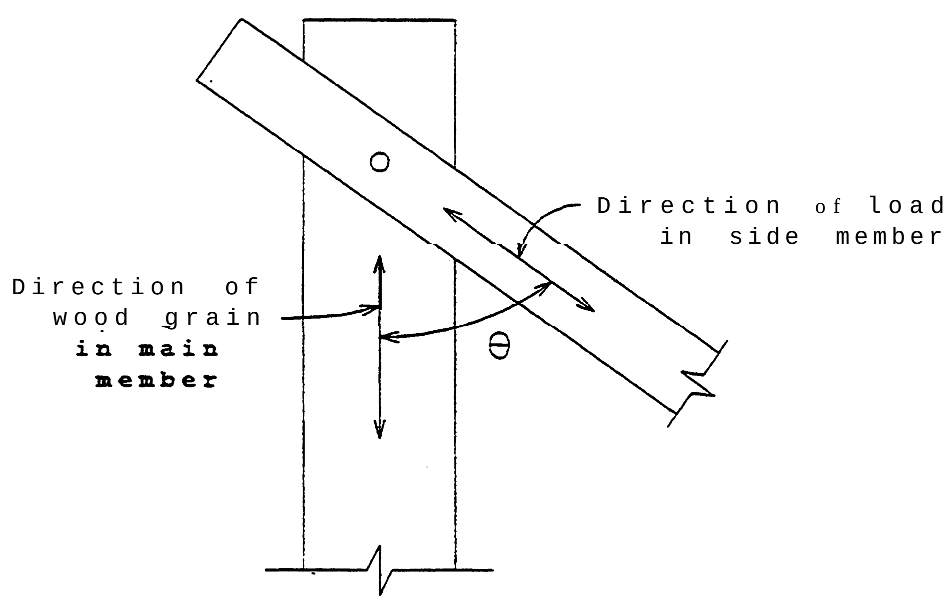



**FIGURE 4-3**

The design values in Table E-l are based on square posts. For a round post, use as the main member thickness the side of a square post having the same cross-sectional area as the round post used.

When appropriately sized steel bars or shapes are used as diagonal bracing, the tabulated design values for main members loaded parallel to the grain (P value) are increased 75 percent for joints made with bolts l/2 inch or less in diameter; 25 percent for joints made with bolts l-l/2 inches in diameter; and proportionally for intermediate diameters. No increase is allowed in the tabulated values for perpendicular-to-grain loading (Q value).

### 4-3.04B Design Procedure

Design values shown in Table E-1 are directly applicable only to three-member joints (bolt in double-shear) in which the side members are each one-half the thickness of the main member. This joint configuration is not typical of bridge falsework where side members are usually much smaller than main members and where two-member joints (bolt in single shear) are common,

Full-scale load tests on bolted connections carried out at the California Department of Transportation research facility revealed that the industry design procedure for two-member joints, which assumes a single-shear load factor of 0.50, is overly conservative for falsework members. A factor of 0.75 was found to be a more realistic value for falsework requirements. In view of this, the procedure adopted by the Division of Structures uses a shear factor of 0.75 when evaluating the adequacy of two-member bolted connections as explained in the following section.

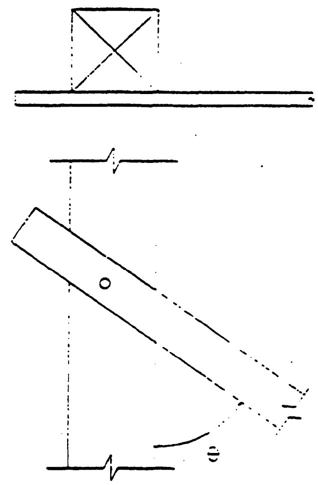
### 4-3.04B(l) Two-Member Connections

Figure 4-4 shows a typical two-member bolted connection in which the side member is loaded parallel-to-grain and the main member is loaded at an angle to the grain.

For a two-member connection, the connector design value is the lessor of the design values for the side and main member determined in accordance with the following:

- For the side member, the design value is three-fourths of the tabulated design value (TDV) for a piece twice the thickness of the side member. To find the tabulated design value, enter Table E-1, Column P, with a member twice the thickness of the side member.

SIDE MEMBER DESIGN VALUE = (0.75)(TDV)



TWO-MEMBER BOLTED CONNECTION

**FIGURE 4-4**

- For the main member, the design value is three-fourths of the value obtained from the Hankinson formula using the tabulated parallel-to-grain (P value) and perpendicular-to-grain (Q value) design values in Table E-1 for a piece the thickness of the main member.

$$\text{MAIN MEMBER DESIGN VALUE} = \frac{(0.75)PQ}{P\sin^2\theta + Q\cos^2\theta}$$

where P and Q are, respectively, the tabulated design values for a load applied parallel to and perpendicular to the grain, and $\theta$ is the angle between the main and side members.

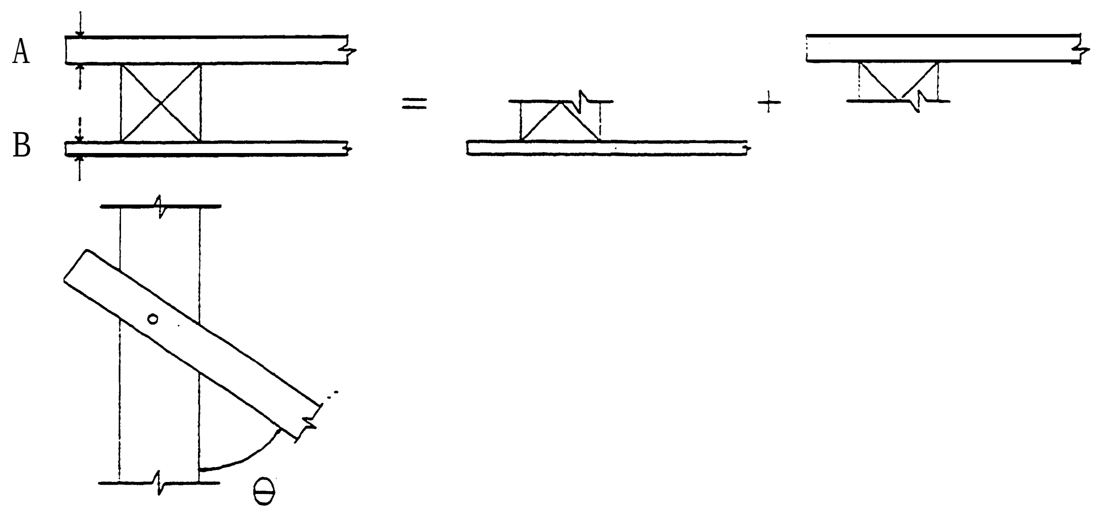
The design procedure discussed in the preceding paragraphs is valid only when the direction of the load is perpendicular to the axis of the bolt. If the load acts at an angle to the bolt axis, as will be the case with longitudinal bracing when false-work bents are skewed, the design values must be determined using a single-shear factor of 0.50, rather than the 0.75 factor used when the load is perpendicular to the bolt axis. For this condition, the design value formulas become:

$$\text{MAIN MEMBER DESIGN VALUE} = (0.5)(\text{TDV})$$

$$\text{MAIN MEMBER DESIGN VALUE} = \frac{(0.50)PQ}{P\sin^2\theta + Q\cos^2\theta}$$

## 4-3.04B(2) Three-Member Connections

Figure 4-5 shows a three-member connection in which the side members are loaded parallel-to-grain and the main member is loaded at an angle to the grain. In a three member connection, each side member connection functions as an independent two-member connection. The allowable connector design value for each side member is determined individually in accordance with the previously described procedure for two-member connections.




THREE-MEMBER BOLTED CONNECTION

**FIGURE 4-5**

## 4-3.04C Installation Requirements

Although installation is primarily a construction concern, the design review should verify that bolt installation will meet the following industry criteria for end and edge distance:

- For parallel-to-grain loading, the minimum end distance for full design load shall be:

    (a) In tension, 7 times the bolt diameter.

    (b) In compression, 4 times the bolt diameter;

- For perpendicular-to-grain loading, the minimum end distance shall be 4 times the bolt diameter.

- For parallel-to-grain loading in tension or compression, the edge distance shall be at least 1.5 times the bolt diameter.

- For perpendicular-to-grain loading, the edge distance toward which the load is acting shall be at least 4 times the bolt diameter and the distance on the opposite edge shall be at least 1.5 bolt diameters. When load reversal occurs, as will be the case for diagonal bracing in timber frames, 4 bolt diameters will be needed at both edges.

See Chapter 9 for additional information on fabrication of bolted connections and installation of bolts.

4-3.04D Multiple-Bolt Connections

When two bolts are used in a connection, the total connector capacity is the sum of the design values of the two individual bolts.

When two bolts are used, they must be set on the axis of the side member. The minimum distance (spacing) between the bolts is 4 times the bolt -diameter, measured center-to-center of the bolt holes.

When more than two bolts are used to connect timber members, industry guidelines impose additional requirements that must be followed in the design. These guidelines, to the extent they are applicable to falsework construction, are discussed in Appendix E. Appendix E also includes example problems showing the design procedure for multiple-bolt installations.

4-3.05 Lag Screw Connection [6]

Design values for lag screws for both withdrawal and lateral loading may be found in Tables E-2 and E-3 in Appendix E.

The tabulated values apply only when the lag screw is installed in a properly sized predrilled hole. (See Chapter 9.)

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[6]As a point of interest, note that lag screws, or lag bolts as they are sometimes called, are not "bolts" in the commonly understood meaning of the term. Lag screws are pointed on the end opposite the head and have a screw-type thread. In a lag screw connection, the lag screw penetrates into but not through the main member. Bolts have a constant diameter and are uniformly threaded to receive a nut. In a bolted connection, the bolt will extend through all members. Because of their superior performance characteristics, bolts are assigned a much higher fastening value than lag screws of the same nominal diameter.

For withdrawal resistance, the tables show the withdrawal value in pounds per inch of penetration of the threaded part of the lag screw into the side grain of the member holding the point, with the axis of the screw perpendicular to that member.

For end-grain withdrawal, the allowable design value may not exceed 75 percent of the corresponding value for withdrawal from side grain.

For lateral load resistance, the tables list design values for loads applied both parallel and perpendicular to the direction of the wood grain. When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value is obtained from the Hankinson formula. (See discussion in Section 4-3.04A, Design Values.)

The tabulated lateral load design values apply only when the lag screw is inserted into the side grain of the member holding the point. If for a particular use, a lag screw is inserted into the end grain of the main member, the design value is 2/3 of the value shown for a load acting perpendicular to-the grain (the Q value).

Industry standards require the spacing, edge and end distances, and net section for lag screw connections to conform to the requirements for bolted connections made with bolts having a diameter equal to the shank diameter of the lag screw.

### 4-3.06 Drift Pin and Drift Bolt Connections

Drift pins are steel rods cut to any desired length. Drift bolts are steel rods manufactured with a bolt head on one end. Typically, these fasteners are used to connect large members, such as caps and posts in a timber bent.

When drift pins or drift bolts are used at locations where the connection is subject to analysis, the required penetration will be determined as provided in this section.

#### 4-3.06A Lateral Resistance

In accordance with industry standards, the lateral resistance design value of a drift pin or drift bolt inserted into the side grain of a wood member may not exceed 75 percent of the design value for a common bolt of the same diameter and length in the main member.

The timber industry has not established design values for drift pins or bolts inserted into the end grain of a member, as would be the case for a falsework cap-to-post connection. For this type of connection, Division of Structures policy limits the lateral load-resisting capacity to 60 percent of the allowable side grain load (perpendicular to grain, or Q value) for an equal diameter bolt with nut. To develop this strength the drift pin or bolt should penetrate at least 12 diameters into the end grain.

4-3.06B Withdrawal Resistance

Withdrawal resistance is a function of several factors, such as the diameter of the drift pin or bolt, the length of penetration into or through a member, and the density of the wood. However, the timber industry has not adopted specific design values for withdrawal. Rather, the recognized industry standard is that drift pins or drift bolts subject to withdrawal loading are to be "...designed in accordance with good engineering practice? (From *National Design Specification for Wood Construction,* 1991 Edition.)

In the absence of industry-wide criteria, the following formulas developed by the U.S. Forest Products Laboratory may be used to estimate the withdrawa. resistance of drift pins and bolts. The formulas shown are applicable to Douglas-Fir Larch.

For withdrawal from side grain:

$$P = 186D^{3/4}$$

For withdrawal from end grain:

$$P = 85D^{3/4}$$

In the formulas, P is the allowable withdrawal load in pounds per inch of penetration, and D is the diameter of the drift pin or drift bolt.

Values obtained from the formulas are for normal duration of load, and should be increased pursuant to the provisions in Section 4-3.07, Adjustment for Duration of Load.

For installations where the penetration of a drift pin or drift bolt is into the end grain of the holding member and through the side grain of the member being held, as is typical of a cap and post or pile connection, the penetration into the end grain should provide a withdrawal resistance sufficient to develop the side grain withdrawal resistance of the member being held.

4-3.07 Adjustment for Duration of Load [7]

Design values shown in Tables E-l through E-8 are for normal load durations and may be increased for short-duration loading.

Selecting the proper duration-of-load factor to use in the calculations is a matter of engineering judgment. Reference to the duration of load graph in Figure 4-6 indicates that a factor of 1.25 will be applicable to most falsework designs, since falsework is seldom subjected to maximum loading for more than seven days. For loads of shorter duration, such as wind, a larger factor would be appropriate. For stage construction where the falsework will remain loaded for an appreciable length of time, a lower factor may be appropriate.

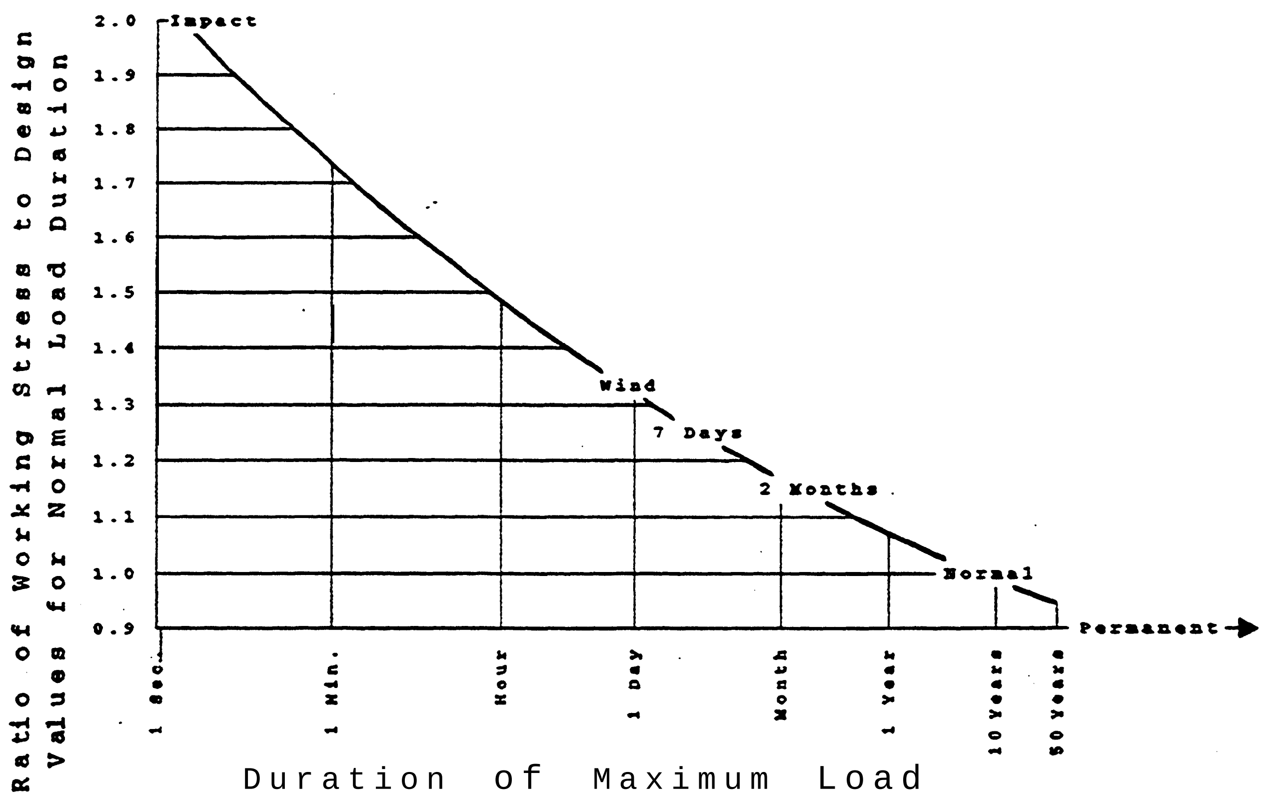



FIGURE 4-6

[7] The adjustment for duration of load as discussed in this section applies only to design values for timber connectors, such as nails, bolts and lag screws. Allowable stresses for. timber and structural steel components used in the connection, as set forth in the specifications, are maximums and thus may not be increased even though the actual duration of load in a given situation may be less than the assumed duration

For normal falsework construction, Division of Structures policy. will permit the use of the following duration-of-load factors:

- a factor of 1.25 when the falsework design is governed by the minimum load (two percent of the design dead load),
- a factor of 1.33 when the design is governed by the wind load.
- a factor of 2.0 (impact loading) for design of the sill-to-base, post-to-sill, cap-to-post and stringer-to-cap connections at traffic openings. (See Chapter 8.)

If the falsework will remain loaded for a relatively longer period, such as cast-in-place prestressed construction where stressing will be delayed or stage construction sequences for any type of concrete structure, the use of a smaller duration of load factor may be appropriate.

## Section 4-4 Steel Members

### 4-4.01 Allowable Stresses

The maximum allowable stresses and other design criteria in the specifications are based on the assumed use of structural steel conforming to ASTM Grade A36. This assumption is reasonable for beams and other sections commonly used in falsework construction, including unusual sections such as old railroad car beams and beams salvaged from dismantled bridges or buildings, because these older sections were rolled from Grade A7 steel. The physical properties of former Grade A7 steel are similar to the properties of the Grade A36 steel being produced today.

The specifications permit the use of higher working stresses for other grades of steel for all loading conditions except flexural compression, provided the grade of steel can be identified. Identification is the contractor's responsibility, subject to verification by the engineer. (See Chapter 9.)

Although the specifications allow higher working stresses when higher strength steels are used, it should be noted that the load-carrying capacity of steel beams will, in most cases, be-limited by deflection, not stress. When deflection controls, the use of high-strength steel will not be on any benefit since the limiting deflection cannot be increased.

High-strength steel beams may provide a greater load-carrying capacity in situations where beams are subjected to bi-axial bending.

## 4-4.02 Bending Stresses

The bending stress formulas are:

$$f = Mc/I \text{ or } f = M/S$$

where f is the bending stress, M is the bending moment, c is the distance from the neutral axis to the extreme fiber, I is the. moment of inertia of the beam about the neutral axis, and S is the beam section modulus.

If the compression flange is supported, these formulas may be used to determine the section needed to carry the applied load for a beam in which bending occurs in the vertical plane only. However, bridge falsework differs from most other construction in that falsework beams are usually set perpendicular to a supporting cap, and the cap is placed parallel to the bridge soffit rather than level. This construction configuration causes the beam to deviate from a true vertical plane by an angle which is equal to the soffit cross slope, and the result is bi-axial bending in the beam. Bi-axial bending is discussed in the following section.

If the compression flange of a beam is not supported, the maximum allowable bending stress must be reduced to prevent flange buckling. (See Section 4-4.04, Flange Buckling, for additional information.)

## 4-4.03 Bi-Axial Bending

Figure 4-7 shows a beam set on a sloping surface. For such beams, the theoretical centroid of the applied load P acts on the top flange through the projected centerline of the web, rather than through the center of gravity of the canted beam. When a vertical load is applied to a canted beam, the load is divided into two components: one acting through the web and one acting along the top flange. This loading condition produces bi-axial bending (i.e., bending in two planes) which decreases the load a given beam is able to carry. The decrease in beam capacity is a function of beam shape and soffit cross slope, and it cannot be determined by inspection. Accordingly, the effect of bi-axial bending must be investigated in all cases where falsework beams are not set in a true vertical plane.

When a beam is set on a sloping surface, the load component acting along the top flange causes the flange to deflect in the downslope direction. For nominal cross slopes, this lateral deflection is small and may be neglected. As the cross slope increases, however, the lateral deflection increases as well, and eventually becomes a factor for consideration since it can adversely affect both form alignment and reinforcing steel clearances.

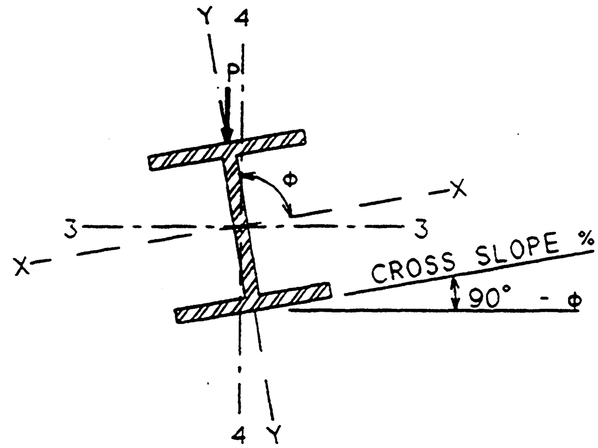



**FIGURE 4-7**

For analysis of bi-axial bending, Division of Structures policy provides for consideration of lateral deflection in all cases where the falsework beam is canted more than 2 percent.

Example Problem 16 in Appendix D illustrates the procedure for calculating stresses and deflections in beams subject to bi-axial bending.

4-4.03A Beams Canted 2 Percent or Less

Refer to Figure 4-7 and note that for any beam subject to bi-axial bending, the maximum bending stress ($f_b$) is the sum of the bending stresses in the x and y directions. The following formulas may be used to calculate $f_b$ based on the moments of inertia of the non-rotated section and the rotation angle $\phi$.

$$f_b = M\left(\frac{y}{I_{XX}}\sin\phi + \frac{x}{I_{YY}}\cos\phi\right)$$

$$I_3 = I_{XX}\sin^2\phi + I_{YY}\cos^2\phi$$

$$I_4 = I_{XX}\cos^2\phi + I_{YY}\sin^2\phi$$

Calculate the actual vertical deflection by using the moment of inertia about the 3-3 axis (I,) in the deflection equation. As previously noted, for this case the lateral deflection may be neglected.

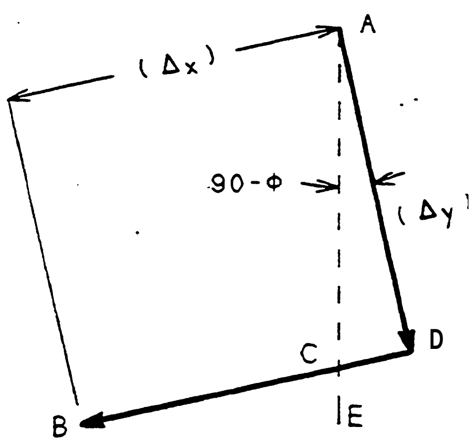
As an alternative procedure, stresses and deflections may be determined with respect to the strong X-X and weak Y-Y axis by using the X and Y components of the applied load P. (See the example problem in Appendix D.)

## 4-4.03B Beams Canted More Than 2 Percent

The maximum bending stress and vertical deflection are computed in accordance with the procedure for beams canted 2 percent or less as discussed in the preceding section. In addition, for box girder structures, it is necessary to evaluate the effect of lateral deflection as discussed herein.

For box girder structures, the net lateral deflection of false-work beams under the weight of the bottom slab and girder stems only is limited to 1.5 inches. This policy limitation is considered necessary to mitigate the adverse affect of lateral movement during the soffit and girder stem concrete pour on form alignment and reinforcing steel placement and clearances.

Refer to Figure 4-8, which is a schematic depiction of the. movement of a point (Point A) on the top flange of a beam which is subject to bi-axial bending as it deflects under load.




AD = vertical deflection = $\Delta_Y$

DB = lateral deflection = $\Delta_X$

CB = net lateral deflection
= DB - DC

**DC = AD(tan90° - $\phi$)**

**FIGURE 4-8**

The movement of point A to point B depicted in Figure 4-8 is the combined vertical and lateral deflection of the bottom slab and stems of a box girder structure as the concrete is placed. While the vertical deflection can be compensated for by camber strips, the lateral deflection DB will displace the bottom slab and stems from the planned alignment (line AE) by the distance CB, which is the net lateral deflection. The net deflection is limited to 1.5 inches maximum.

For the lateral deflection calculation, the vertical load is the dead load weight of the concrete in the soffit slab and girder stems. Use the component values of the vertical load to determine beam deflections about both the X-X and Y-Y axis.

## 4-4.04 Flange Buckling

The compression flange of a beam may be visualized as a column loaded along its length by increments of load transferred to it by horizontal shear from the web. If the compression flange is narrow compared to the depth of the beam, the flange may fail by buckling in somewhat the same manner as a slender column.

Although methods for determining the critical buckling stress are quite complex, in steel members many simplifications are possible. Generally, the empirical formulas used are similar to column formulas, except that the flange width "b" is used instead of the radius of gyration.

The formula in our specification is:

$$f\ (\text{maximum}) = \frac{12{,}000{,}000}{Ld/bt}$$

but not more than 22,000 psi for unidentified steel or steel conforming to ASTM Designation A36, nor more than $0.6F_y$ for other identified grades of steel where $F_y$ is the minimum yield stress.

In the formula, L is the laterally unsupported length, d is the depth of the member and bt is the area of the compression flange. All dimensions are in inches. This formula gives the maximum allowable fiber stress in the beam. If the actual stress exceeds the allowable, the flange must be supported or the load reduced.

It is difficult to determine how much lateral support may be developed by other falsework members. For example, friction between the joists and top flange of a beam will provide some restraint, but the amount is indeterminate. As a matter of policy, therefore, friction between the joists and top flange will be neglected when investigating flange buckling.

Since it is impossible to predict the direction in which the compression flange will buckle, it is necessary to provide positive restraint in both directions. This is an important and often overlooked point. For example, the use of a tension tie between two adjacent beams or welding a light structural steel shape (such as a bar or angle) across the top flange of several beams will not prevent lateral movement. Such measures merely assure that all beams deflect in the same direction. Even when tension ties are used in combination with a compression strut, lateral restraint is not-effective because the restraining components cannot assure that the beams will act as a unit when a lateral force is applied.

Timber cross-bracing between adjacent beams is widely-used for flange support in falsework construction. In this method timber struts, in pairs, are set diagonally between the top flange of one beam and the bottom flange of the adjacent beam to form an X, and securely wedged into place. However, timber cross-bracing alone will not prevent flange buckling because timber struts can resist only compression forces.

Perhaps the most effective flange support method uses steel tension ties welded, clamped or otherwise secured across the top and bottom of adjacent beams in combination with timber cross-bracing between the beams, as shown in Figure 4-9.

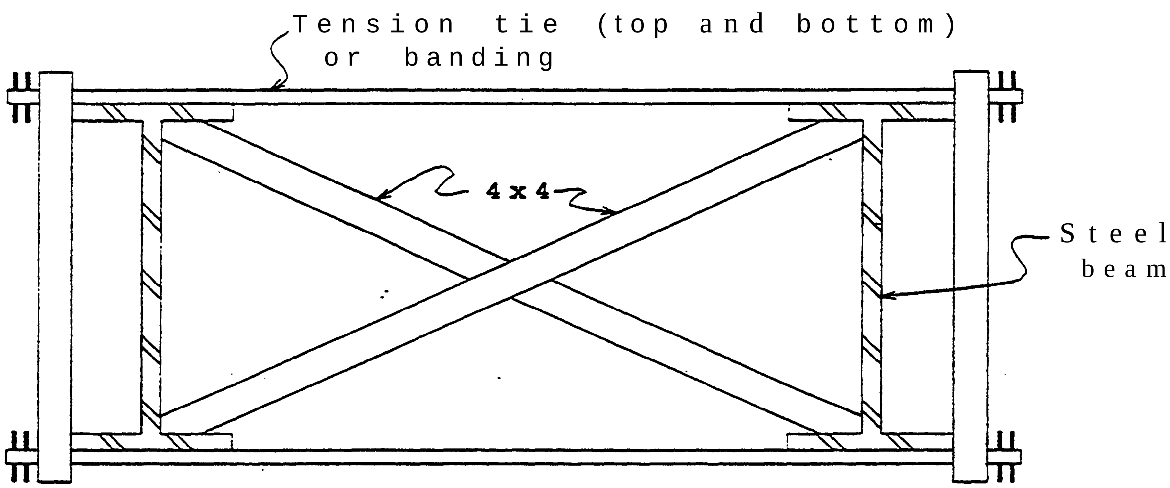



**FIGURE 4-9**

Many contractors use commercial steel banding material wrapped around pairs of adjacent cross-braced beams. Steel banding is less expensive and easier to install and remove than other types of tension components, but banding is not effective unless it is properly installed and tightened. Furthermore, when banding is used as part of a flange support system, some means must be provided to prevent an abrupt bend or kink at the point of contact with the outer edge of the beam flange. This is an important consideration because any kink or sharp bend in commercial banding is, potentially, a point of stress concentration which can reduce the strength of the banding material. The use of softeners will reduce this stress concentration.

Keep in mind that steel banding is a commercial product. If there is any question as to the adequacy of banding installed in a given situation, the contractor should be required to furnish manufacturer's catalog data and instructions for use.

Bracing, blocking, steel banding, ties, etc., required for lateral support of beam flanges must be installed at right angles to the beam. Bracing in adjacent bays should be set in the same transverse plane, if possible. If, because of skew or other considerations, it is necessary to offset the bracing in adjacent bays, the offset distance should not exceed twice the depth of the beam.

Finally, when investigating flange buckling, keep in mind that only a small force is needed to hold the compression flange in. position. In steel design for permanent work, it is common practice to use an assumed value of two percent of the calculated compression force in the beam flange at the point under consideration as the design force for the supporting brace, and this practice will be acceptable for bridge falsework.

Providing adequate flange support, when support is necessary to prevent overstressing the compression flange, is an important design consideration. The method of support, including all construction details, must be shown on the falsework drawings.

### 4-4.05 Beam Shear

The shearing stress at any point in a steel beam is calculated from the general formula for shearing stress. The general formula is:

$$v = VQ/Ib$$

where v is the shearing stress on any given horizontal section or plane through the beam, V is the vertical shear, Q is the statical moment about the neutral axis of that portion of the beam cross-section-which is outside of the plane where the shearing stress is wanted, I is the moment of inertia of the entire cross-section, and b is the width of the beam at the point under consideration.

Since the web of a steel beam resists most of the shear, the shearing stress is usually checked by the formula:

$$v = V/ht$$

where h is the overall depth of the beam (out-to-out of the beam flanges) and t is the web thickness.

If a shearing stress occurs in one plane, an equal shearing stress occurs in a plane through that point perpendicular to the first plane. Therefore, the shear formula may be used to determine both vertical and horizontal shearing stresses.

## 4-4.06 Web Crippling

Rolled beams should be checked to see that the end reaction and/or any concentrated load along the interior of the beam does not produce a compressive stress at the web toe of the fillet in excess of the allowable stress of 27,000 psi.

The following formulas are used to check web crippling:

For end reactions,

$$f = \frac{R}{(a+k)t_w}$$

For interior loads,

$$f = \frac{R}{(a+2k)t_w}$$

In the formulas, f is the stress, in psi; R is the concentrated load or end reaction, in pounds; $t_w$ is the web-thickness; k is the distance from the outer face of the flange to the web toe of the fillet; and a is the length of bearing, but not less than k for end reactions. All dimensions are in inches.

If the actual value exceeds the allowable, the web must be stiffened or the length of bearing increased,

When rolled beams require bearing stiffeners to prevent web crippling, the stiffeners may be designed to resist only the portion of the total load that is in excess of the load the beam is capable of resistitig without stiffeners.

## 4-4.07 Steel Posts and Columns

The specified column formula limits the design axial load to that which will not produce an average unit stress exceeding

$$\frac{P}{A} = 16,000 - 0.38\left(\frac{L}{r}\right)^2$$

In the formula, r is the radius of gyration of the section and L is the unsupported length. The limiting L/r value is 120.

In a column with pinned ends and no intermediate support, L is the actual length. Columns with other end conditions require the use of an effective length instead of the actual length. The effective length is the length of column which actually behaves as though it were pinned.

Determining the effective length of a column with restrained end conditions is an unnecessary refinement for falsework design. Accordingly, it is accepted practice to treat columns in falsework bents as though they were pin-connected, which is conservative for columns with end restraint.

### 4-4.08 Steel Bracing

For bolted connections, the bolt design values may be taken from the AISC *Manual of Steel Construction,* In accordance with AISC design practice, the calculated bearing stress on the projected area of the bolt in steel members may not exceed 1.35 times the specified yield strength of the steel. For A36 Grade material, the allowable bearing stress is 48,600 psi. This value may not be increased for falsework construction.

The strength of fillet-welded connections may be approximated by assuming a value of 1000 pounds per inch for each l/8-inch of fillet weld. While this value would be considered conservative for permanent work where welding is performed under controlled conditions, it is realistic for the welding procedures commonly used for falsework construction.

If the design of the connection is based on a higher weld value, welding must conform to the quality standards associated with permanent construction. (See Field Welding in Chapter 9.)

Structural steel components (angles, bars, etc.) are sometimes used as diagonal bracing in timber bents. In such cases, the bolt design values for parallel-to-the-grain loading in the main member may be increased as discussed in Section 4-3.04A, Design Values, No increase is allowed for perpendicular-to-the-grain loading, however.

### 4-4.09 Miscellaneous Steel Components

The adequacy of miscellaneous components (such as anchor bolts, column base plates, grillages, hangers, tie bars and similar steel components) and hardware items, when used at locations subject to analysis but not specifically covered by the Falsework Manual, will be determined in accordance with applicable design procedure or recommended practice included in the AISC *Manual of Steel Construction.*

## Section 4-5 Cable Bracing Systems

### 4-5.01 Introduction

As used in this manual, the term "cable bracing system" means a length of wire rope cable and an appropriate fastening device. Cable bracing systems may be used to resist both overturning and collapsing forces. (See Chapter 5 for a discussion of overturning and collapse as falsework failure modes.)

Cable systems are particularly effective in resisting the overturning of high falsework, and when used for this purpose they are relatively inexpensive as compared to other bracing methods. Cable is also used extensively as temporary bracing to stabilize falsework bents while they are being erected and/or removed.

Less common is the use of cables as diagonal bracing to resist internal collapse of a falsework bent. Design of cable systems to resist internal collapse is a highly sophisticated exercise, particularly with respect to such factors as the anticipated cable elongation, the amount of preloading or cable tension needed, the effect of cable tension on other falsework members, and similar factors which affect system stability.

Division of Structures policy with respect to the use of cable bracing systems, and the procedures and methodology to be used by field engineers when reviewing the adequacy of cable designs, are discussed in the following sections.

### 4-5.02 Required Information for all Cable Systems

When cable bracing is used, the cables and cable fastening devices are an essential part of the falsework design. All elements of the cable bracing system must be shown on the falsework drawings in sufficient detail *to* permit a stress analysis. In addition, the contractor must provide technical data showing the strength and physical properties of the cable to be used. (See Manufacturer's Technical Data and Required Cable Design Data in Sections 4-5.03 and 4-5.07B, respectively.)

The following information is to be shown on the falsework drawings for all cable systems:

- The cable diameter and, for internal bracing systems, the preload value.
- The type and number of fastening devices (such as Crosby clips, plate clamps, etc.) to be used at each connection.

- The method by which the cables may be tightened after installation, if tightening is necessary to ensure their continued effectiveness.

- For cables designed to resist overturning forces, the cable anchorage must be shown. (See Section 4-5.09, Cable Anchor Systems.)

- The location and method of attachment of the cable to the falsework must be shown. This is a particularly important feature of the design, since the connecting device must transfer both horizontal and vertical forces to the cable without overstressing any falsework component. [8]

4-5.03 Manufacturer's Technical Data

For application of the falsework specifications, a cable bracing system (i.e., the cable together with cable fastening devices) is a manufactured assembly. Accordingly, the cable must be installed in accordance with the manufacturer's recommendations, and the contractor must (if requested) furnish manufacturer's technical data.

For all cable systems, technical data from the manufacturer must include either the breaking strength (which may be identified as a rated or nominal strength) or the safe working load of the cable, the cable diameter, and enough descriptive information (number of strands, number of wires per strand, core type, etc.) to identify the cable in the field. Additional information is needed when cable is used as diagonal bracing to prevent the collapse of a falsework bent. (See Section 4-5.07, Review Criteria for Internal Cable Bracing Systems.) Manufacturer's technical data is also required for all cable fastening devices (Crosby clips, turnbuckles, shackles, etc.)

Since the adequacy of a cable bracing design cannot be verified without reference to the technical data provided by the cable manufacturer, such data is an essential part of the falsework drawing submittal. In any case where falsework drawings showing a cable bracing system are not accompanied by technical data from the cable manufacturer, the contractor should be informed immediately. Pursuant to Division policy, any falsework drawing submittal requiring technical data is not complete until such data is provided. (See Chapter 2.)

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[8] When cables are used to prevent overturning of heavy duty shoring, cable restraint must be designed to act through the cap system. Cables should not be attached to *tower* components unless the contractor has obtained written authorization from the shoring system manufacturer. Such authorization must be furnished before the drawings are approved. (See Chapter 6.)

## WIRE ROPE CONNECTIONS

**(As compared to Safe Loads on Wire Rope)**

| Figure | Type of Connection | Efficiency |
|---|---|---|
| | Wire Rope | 100% |
| 2 | Sockets - Zinc Type | 100% |
| 3 | Wedge Sockets | 70% |
| 4 | Clips - Crosby Type | 80% |
| 5 | Knot and Clip (Contractors Knot) | 50% |
| 6 | Plate Clamp - Three Bolt Type | 80% |
| 7 | Spliced Eye and Thimble: | |
| | I/4" and smaller | 100% |
| | 3/8" to 3/4" | 96% |
| | 7/8" to 1" | 88% |
| | I-1/8" to I-1/2" | 82% |
| | 1-6/8" to 2" | 75% |
| | 2-I/8" and larger | 70% |

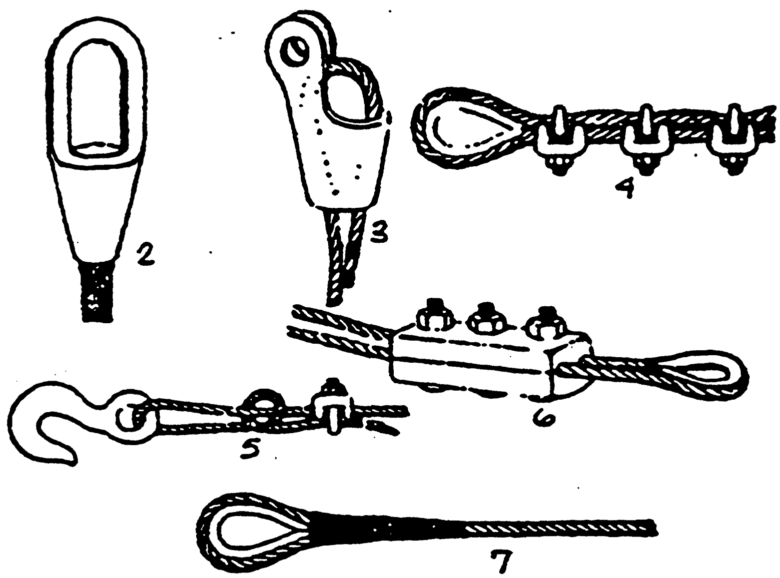



**TABLE 4-1**

In the absence of technical data, a load test will be required to establish cable strength and physical properties. (See Section 4-5.08, Cable Load Tests.)

### 4-5.04 Cable Connector Design

Cable connectors shall be designed in accordance with criteria shown in Table 4-1 and Table 4-3. Connector efficiency assumed in the design shall not exceed the values shown in Table 4-l.

If U-bolt (Crosby type) wire rope clips are used as connectors,, the number used and the spacing must conform to the data shown in Table 4-3, and must be shown on the drawings. Note that forged clips have somewhat greater holding strength. Forged clips are marked "forged" to permit positive identification, and have the appearance of galvanized metal. Malleable cable clips appear smooth and shiny.

### 4-5.05 Cable Elongation

Wire rope cable is an elastic material; consequently, it will elongate or stretch when loaded. However, cable is a unique elastic material in that its elongation is not uniform throughout the elastic range. Additionally, it is subject to inelastic deformation at loads well below the yield strength.

For descriptive purposes, the cable industry identifies the two properties that contribute to the total elongation experienced by a cable during its service life as "elastic" stretch and "constructional" stretch.

In general, cable elongation will not be a falsework design consideration for cable used as 'external bracing to prevent overturning of the falsework system or a system component. However, to ensure system stability, cable elongation must be considered when cable is used as internal bracing to prevent frame collapse, as discussed in Section 4-5.07, Review Criteria for Internal Cable Bracing Systems.

### 4-5.06 Review Criteria for External Cable Bracing Systems

### 4-5.06A Cable Design Load

When cables are used as external bracing to resist overturning, the horizontal design load to be resisted by the cable bracing system will be calculated as follows:

- When used with heavy-duty shoring, cable bracing shall be designed to resist the difference between the total overturning moment and the resistance to overturning provided by the individual falsework towers. (See Chapters 3 and 5 for overturning considerations.).

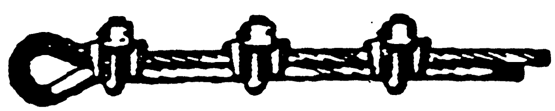
## WIRE ROPE CAPACITIES

**Safe Load in Pounds for New Improved Plow Steel Hoisting Rope**
6 strands of 19 wires. Hemp Center

(SAFETY FACTOR Of 6)

| DIAMETER INCHES | WEIGHT LBS/FT | SAFE LOAD LBS. | DIAMETER INCHES | WEIGHT LBS/FT | SAFE LOAD LBS. |
|---|---|---|---|---|---|
| l/4 | .10 | 1.050 | 1 | 1.60 | 15.000 |
| 5/16 | .16 | 1.500 | 5-1/8 | 2.03 | 18.600 |
| 3/8 | .23 | 2.250 | l-l/4 | 2.50 | 23.000 |
| 7/16 | .31 | 3.070 | l-3/8 | 3.03 | 26.900 |
| 1/2 | .40 | 4.030 | 1-1/2 | 3.60 | 30.700 |
| 9/16 | .61 | 4.840 | l-6/8 | 4.23 | 36.700 |
| 6/8 | .63 | 6.330 | 1-3/4 | 4.90 | 41,300 |

**TABLE 4-2**



## APPLYING WIRE ROPE CUPS

The only correct methnod of attatching U-bolt wire rope clips to rope ends is shown in the illustration. The base (saddle) of the clip bears against the live end of the rope, while the "U" of the bolt presses against the dead end. A useful method of remembering this is: "You never saddle a dead horse."

The clips are usually spaced about six rope diameters apart to give adequate holding power. A wire-rope thimble should be used in the loop eye to prevent kinking when wire rope clips are used. The correct number of clips for safe application, and spacing distances, are shown in the table below.

NUMBER OF CLIPS AND SPACING FOR SAFE APPLICATION

| Improved Plow Steel Rope Diameter Inches | Number of Clips Drop Forged | Number of Clips Other Material | Minimum Spacing (inches) |
|---|---|---|---|
| 3/8 | 2 | 3 | 3 |
| 1/2 | 3 | 4 | 3-1/2 |
| 5/8 | 3 | 4 | 4 |
| 3/4 | 4 | 5 | 4-1/2 |
| 7/8 | 4 | 5 | 5-1/4 |
| 1 | 5 | 6 | 6 |
| 1-1/8 | 6 | 6 | 6-3/4 |
| 1-1/4 | 6 | 7 | 7-1/2 |
| 1-3/8 | 7 | 7 | 8-1/4 |
| 1-1/2 | 7 | 8 | 9 |

**TABLE 4-3**

- When used with pipe-frame shoring, cable bracing shall be designed to resist the difference between the total overturning moment and the resistance to overturning provided by the shoring system as a whole. (See Chapter 6 for analysis of pipe frame shoring systems.)

- When used as external bracing to prevent overturning of all other types of falsework, including temporary bracing used to stabilize falsework components during erection and/or removal, cable bracing shall be designed to resist the total overturning moment.

4-5.06B Determining Allowable Working Loads

The maximum allowable load to be carried by a given cable will be determined in accordance with the following criteria:

- If the cable is new or in uniformly good condition, and if it can be identified by reference to a manufacturer's catalog or similar technical publication, the allowable cable load will be the breaking strength-of the cable as shown in the manufacturer's catalog, multiplied by the efficiency of the cable connector, and divided by a safety factor of 2. (That is, allowable working load = breaking strength x connector efficiency ÷ safety factor.)

  While the technical data provided by the manufacturer will in most cases show the breaking (or nominal) strength of the cable; some manufacturer's catalogs show only a recommended safe working load. Should this be the case, the cable design load may not exceed the safe working load value, unless the contractor elects to perform a load test.[9]

- If the cable is used and still in serviceable condition, but a manufacturer's catalog is not available, the contractor may elect to perform a load test. In such cases, the allowable working load shall not exceed the breaking strength as determined by the load test, multiplied by the connector efficiency factor, and divided by a safety factor of 2.

---

[9] It is important to note that, for a given cable, the safe working load recommended by the manufacturer will be considerably less than the allowable load determined from the breaking strength of the cable. This is the case because the recommended safe working load will be based on a safety factor of 5:l or more in consideration of the dynamic loading to which cable is ordinarily subjected, rather than the 2:l safety factor which is appropriate for the static loading condition associated with falsework construction.

- If the cable is used and still in serviceable condition, and if the contractor does not perform a load test, the allowable working load shall not exceed the wire rope capacity shown in Table 4-2 multiplied by the cable connector efficiency.

## 4-5.07 Review Criteria for Internal Cable Bracing Systems [10]

### 4-5.07A Limitations and Conditions of Use

Division of Structures policy limits the use of internal cable bracing systems to single tier falsework bents where the cable is fastened to the bent cap and sill beam. The use of cable to provide frame rigidity in multi-tiered bents, or in any bent where the cable is attached to a post or column, will not be permitted. (See Chapter 5 for definitions of single and multi-tiered bents.)

For analysis of single tier cable systems, Division policy limits the calculated horizontal deflection to a maximum value numerically equal to one-eighth inch per foot of post height, but not more than one-fourth of the post width measured along the cap or sill beam, or in the case of pipe columns, not more than one-fourth of the diameter of the pipe column.,

Limiting the horizontal deflection is necessary to prevent undesirable frame distortion, and to ensure that the vertical load reaction remains within the base width of the post or pipe column. The limiting values in the preceding paragraph assure this objective.

The calculated horizontal deflection must be based on cable elongation due to both elastic and constructional stretch, as discussed later in this section.

When used to provide internal stability in a timber bent, the cable must be attached to the timber members with an appropriate fastening device installed in accordance with the manufacturer's recommendations. Because of the need to accurately predict the amount of lateral deflection in the system, Division policy does not permit internal cable bracing to be looped around timber members. [11]

---

[10] As used in this section, the term cable also means and includes prestressing strand when prestressing strand is used in an internal bracing system.

[11] Looping of cables around timber members, while prohibited in internal bracing systems, is an acceptable practice when the cable is used to prevent overturning, such as in longitudinal and/or temporary bracing systems.

## 4-5.07B Required Cable Design Data

When cable is to be used as internal bracing, the technical data accompanying the falsework drawings must include the following information:

- Cable description (including cable nominal diameter, number of strands, number of wires per strand and core type,)
- Weight of the cable.
- Breaking or nominal strength. [12]
- Net metallic area and modulus of elasticity.
- Maximum constructional stretch (percent of loaded length.)

## 4-5.07C Factor of Safety

As discussed elsewhere in this manual, the allowable (or design) load-carrying capacity of a product or device, including cable used to prevent overturning, is obtained by applying a factor of safety based on the ultimate. strength of that-product or device. (See, for example, Section 4-5.06, Review Criteria for External Cable Bracing.) This approach is satisfactory for falsework because, in general, system integrity will not be jeopardized by inelastic deformation that may occur if a product or device is subjected to a load that exceeds its yield strength, provided the load is not greater than the ultimate strength. However, this practice is not appropriate for cable used as internal bracing because of the need to limit the total- cable elongation to a predictable amount. In view of this reality, when cable is used as internal bracing for falsework, the allowable working load must be related to the yield strength of the cable.

While a safety factor of 2 based on yield strength is considered satisfactory for falsework, it is noted that industry standards established by the Wire Rope Technical Board require the-safe working load for static loading conditions to be determined using a safety factor of 3 based on the nominal (breaking) strength of the cable. Since cables of the type used for falsework have a yield strength of approximately 65 percent of the nominal strength, the industry standard is consistent with the use of a safety factor of *2* based on yield strength. In view of this and to simplify the calculations, Division of Structures review methodology uses a safety factor of *3* based on the nominal (breaking) strength when determining the design capacity of the cable units.

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[12] The *Wire Rope Users Manual* uses the term "nominal" strength to describe the maximum load that a given cable may be expected to carry, and this term may be used by the manufacturer as well.

## 4-5.07D Cable Preloading

After assembly, all cable units are to be preloaded to remove any slack in the cable and connections. Preloading is necessary to ensure that the cable units will act elastically when the loads are applied. [13]

Determining whether a given preload force is sufficient to ensure that the cable bracing system will act elastically involves a number of subjective considerations. In the past, an arbitrary value of 1000 pounds has been commonly used as a minimum preload force; however, the actual force needed to remove cable slack is a function of both the length and weight of the cable, and thus there is no single preload value that will be appropriate for all installations. [14]

To verify design adequacy, the Division of Structures uses the relationship between preload force and the theoretical drape of the cable hanging under its own weight. Refer to Figure 4-10 and note that dimension A is the midspan cable drape (i.e., the drape at the midpoint of the horizontal distance between cable connection points.)

For any given preload value, the drape may be calculated using the Figure 4-10 formula. Pursuant to Division of Structures policy, the preload force shown on the falsework drawings must tension the cable unit sufficiently so that the calculated cable drape, after the falsework is erected, will not exceed the following values:

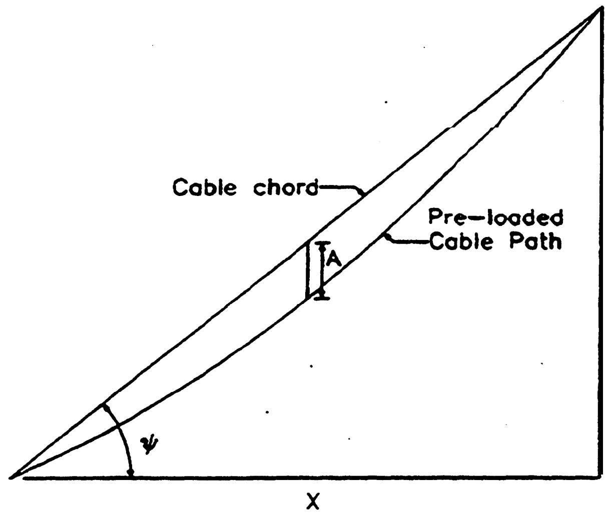
| Cable Size | Maximum Cable Drape (A) |
|---|---|
| 3/8 | 1″ |
| l/2 | 2″ |
| l/2 | 2 3/4" |

Experience has demonstrated that a preload force of less than about 500 pounds may be insufficient to remove all cable slack. Therefore, the minimum preload value shall not be less than 500 pounds, regardless of other considerations.

---

[13] As used in this section, the term "cable unit" refers to all cables acting to resist forces in the same direction, and the term "cable" refers to each individual cable within the cable unit.

[14] This fact should be obvious since the required preload force for large diameter cables used in a high falsework bent will be significantly greater than the preload force required for a bent where the cable length is relatively short and small diameter cable is used.




$$A = \frac{qx^2}{8P\cos\psi}$$

In the equation:

q = cable weight per foot

x= horizontal distance between cable connection points

P = cable preload value

**FIGURE 4-10**

For bents where the top cap and bottom sill are not parallel, as will be the case when the cap is set parallel to the bridge soffit, post height Variation will produce a non-symmetrical cable arrangement wherein the opposing cable units will have different lengths, and thus different elongations under a given preload force. However, the horizontal component of the cable elongation at the top cap connection must be equal in both directions to prevent distortion of the falsework bent. This means that, except for symmetrical cable arrangements, opposing cable units will have slightly different preload values.

Preload values are to be calculated by the contractor and shown on the falsework drawings, along with the method by which the required preload force is to be measured.

4-5.07E Determining Cable Elongation [15]

For a given installation and design load, the total cable elongation is a function of two independent factors. These factors are "elastic" stretch and "constructional" stretch.

Elastic stretch is the result of the inherent elasticity, or recoverable deformation, of the metal itself. Since the elastic properties of a given cable can be determined, elongation due to elastic stretch is predictable.

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[15] The assumptions and design policy discussed herein are based on recommendations and design standards in the *Wire Rope Users Manual,* Third Edition, issued by the Wire Rope Technical Board. Industry recommendations and standards are modified as appropriate for falsework considerations.

Constructional stretch occurs when cable is loaded for the first time. When a cable is first loaded, the helically-wound wires and strands are pulled more tightly together, compressing the core and bringing all of the cable elements into closer contact. This results in a slight reduction in diameter and a corresponding increase in length.

Constructional stretch is influenced by several factors. These include the type of core, the number of strands and the number of wires in each strand, the manner in which the cable is wound, and the magnitude of the applied load.

Because of the number of variable factors involved, there is no mathematical constant applicable to all cable types from which elongation due to constructional stretch may be determined. For a given cable and load, however, the probable constructional stretch can be approximated with sufficient accuracy for cable design considerations.

### 4-5.07E(l) Determining Elastic Stretch

For an elastic material loaded within the elastic range, the elastic deformation (i.e., the change in length, or stretch) is directly proportional to the change in applied load, all other factors remaining equal.

The general formula for elastic deformation is:

$$\Delta = \frac{\text{Change in Load} \times \text{Length}}{\text{Area} \times \text{Modulus of Elasticity}}$$

Unlike other elastic materials, cable elongation is not directly proportional to the applied load over the full elastic range. This is the case because the modulus of elasticity for a given cable is significantly lower at low levels of applied load than at loads nearer to the normal working strength of the cable.

To accommodate this unique physical characteristic, it is standard practice in the cable industry to facilitate cable elongation calculations by using a nominal E value and a reduced E value, depending on the magnitude of the applied load. The nominal E value is used for that portion of the total load which exceeds 20 percent of the breaking strength of the cable. The reduced E value (which is equal to 90 percent of the nominal value), is used for the portion of the load between zero and 20 percent of the breaking strength.

If the cable design load is not greater than 20 percent of the cable breaking strength, the elastic stretch may be determined from the general formula for elastic deformation shown above, using the reduced E value.

If the cable design load is greater than 20 percent of the breaking strength, the total elastic stretch is the sum of $\Delta_1$ and $\Delta_2$ as given by the following formulas:

$$\Delta_1 = \frac{(0.2 \times \text{Breaking Strength} - \text{Preload})\ (L)}{A \times 0.9E}$$

$$\Delta_2 = \frac{(\text{Cable Load} - 0.2 \times \text{Breaking Strength})\ (L + \Delta_1)}{A \times E}$$

In the equations, A values are in feet; L is the loaded length of the cable, in feet; A is the net metallic area of the cable, in square inches; E is the nominal modulus of-elasticity, in pounds per square inch. Breaking strength, preload and cable load values are expressed in pounds.

<u>4-507E(2) Determining Constructional Stretch</u>

As previously noted, constructional stretch occurs when a cable is loaded for the first time. Constructional stretch is an important design consideration for internal cable bracing systems because, depending on cable type, a typical new wire rope cable initially loaded to its design working strength will undergo a permanent elongation of from l/2 to 1 percent of the loaded length.

Industry design practice assumes that constructional stretch is proportional to the applied load, and that all constructional stretch occurs within the elastic range. (That is, the total expected constructional stretch will have occurred when the applied load reaches the yield point load, or 65 percent of the cable breakingstrength.)

Constructional stretch is given by the following formula:

$$\Delta_{CS} = \left(\frac{\text{Applied Load}}{0.65 \times \text{Cable Strength}}\right) (CS\%)\ (L)$$

In the formula, $\Delta_{CS}$ is the constructional stretch, in feet; *CS%* is the constructional stretch, in percent, given by the cable manufacturer; and L is the cable length, in feet, between end connections. Applied load and cable strength are in pounds.

The anticipated constructional stretch, expressed as a percentage will be included in the cable design data provided by the manufacturer. If for some reason it is not provided and cannot be obtained, the analysis may be based on assumed values of 3/4 and 1 percent for wire core and fiber core cables, respectively.

Some types of high strength cable, such as prestressing strand, are commercially available with constructional stretch removed by preloading at the factory. Such cable will conform to the requirements for ASTM Designation A586 (structural strand) or ASTM Designation A603 (structural rope), and will be clearly identified as prestretched cable. When prestretched cable is used, it is not necessary to consider constructional stretch in the analysis. [16]

<u>4-5.07F Procedure for Analysis [17]</u>

When cable is used to prevent internal collapse of a falsework bent, the method used to evaluate the adequacy of the bracing system differs from the traditional methods normally used for frame analysis because of the need to consider-the effect of cable elongation and frame deflection on system stability.

In a cable-braced frame, the cable elongates as the horizontal design load is applied. Cable elongation allows the frame to deflect, producing vertical load eccentricity which must be considered in the analysis. Additionally, post reactions will be affected by the vertical component of the cable load.

---

[16] Note that cable conforming to ASTM Designation A586 or A603 may be either prestretched or non-prestretched. Prestretched cable must be so identified in the cable design data furnished by the manufacturer. If the cable is not clearly identified as prestretched, constructional stretch must be considered in the analysis, even though the cable may otherwise conform to the referenced ASTM specifications.

[17] The Division of Structures methodology for analysis of internal cable bracing systems, as described herein, assumes that the post or column is attached to the cap and sill by an eccentric pinned connection, and that the eccentricity is numerically equal to the horizontal movement of the cap due to cable unit elongation. These assumptions are valid for typical pipe column bents where the connections are not designed to resist moment, and for all timber bents. However, it is theoretically possible to design a pipe column frame with fixed connections. Any such designs will require a rigorous analysis by the contractor (with supporting calculations) and similar review by the engineer. In such cases, the engineer should contact the Sacramento Office of Structure Construction for the procedure to be followed.

The following procedure (illustrated in Example Problem 17) is used to evaluate system adequacy:

1 . Determine cable lengths, post heights, and the vertical distance between the plane of the cable connection at the cap and the plane of the cable connection at the sill beam.

2 . Calculate the horizontal design load. For analysis of cable systems, the horizontal design load is the larger of (1) the wind load acting on the bent or (2) two percent of the total dead load based on the full superstructure cross section.

3. Calculate the capacity of the cable units, using a factor of safety of 3 based on the breaking (nominal) strength of the cable. (See Section 4-5.07C, Factor of Safety:)

4. Check the cable preload values shown on the falsework drawings.

5 . Using the horizontal design load from step 2, calculate the cable unit design load.

6. Compare the cable unit design load from step 5 and the cable unit capacity determined in step 3. If the design load exceeds capacity, the system must be redesigned.

7. Calculate the cable unit elongation, which is the sum of the elongations due to elastic and constructional stretch. [18]

   (a) Calculate the elastic stretch, using the formulas shown in Section 4-5.07E(l), Determining Elastic Stretch.

   (b) Calculate. the constructional stretch. Constructional stretch is expressed as a percent of the loaded length of the cable. For falsework bents, the loaded length is the length between end connections. Follow the procedure explained in Section 4-5.07E(2), Determining Constructional Stretch.

   (c) Add the elastic stretch and constructional stretch to obtain the total elongation for the cable unit.

---

[18] As previously noted, Division of Structures policy requires a consideration of both elastic and constructional stretch when calculating the expected cable elongation, unless the cable to be used has been preloaded at the factory to remove the constructional stretch.

8. Calculate the horizontal displacement of the top cap due to cable unit elongation. For this calculation, all posts are assumed to rotate about their bases, and their tops move laterally the same distance as the cap, as depicted in Figure 4-11.

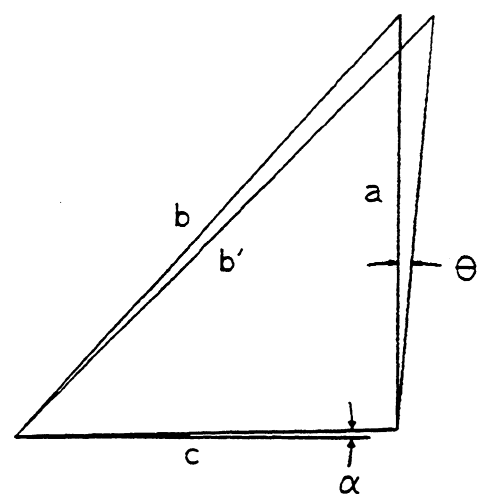


**FIGURE 4-11**

Refer to Figure 4-11 and note that the vertical distance between the cap and sill cable connection points at the location of the cap cable connection (vertical line a) may be used to complete. triangles for the preloaded (b) and fully loaded (b') cables, The law of cosines may be used to determine angles, since the dimensions of all three legs of the triangles will be known. Once the angle of rotation ($\theta$) of the posts has been determined, the horizontal displacement at the tops of the posts can be calculated.

9 . Compare the calculated horizontal displacement and the allowable horizontal displacement.

The calculated horizontal displacement is limited to one-eighth inch per foot of post height, or one-fourth of the post width or diameter, whichever is the smaller value. (See Section 4-5.07A, Limitations and Conditions of Use.) If the calculated horizontal displacement exceeds the allowable displacement, the system must be redesigned.

## 4-5.07F(l) Completion of Analysis for Box Girder Structures

The Standard Specifications provide for an intermediate loading consideration for box girder structures, so that the method of analysis for box girder structures differs from the method of analysis for other structure types.

For box girder structures there are two load combinations, designated in the specifications as Case I and Case II.

Case I. Design live load, plus total design dead load excluding the weight of the concrete in the deck slab, plus total horizontal design load, plus the vertical component of the cable unit design load. For analysis, all vertical loads act on the falsework in its deflected position.

Case II. Design live load, plus total design dead load, plus the total horizontal design load. For analysis, both the live load and the dead load act on the falsework in its Case I deflected position,

The procedure for box girders structures is as follows:

10. Calculate the post loads and the bending moment in the cap and sill beam for both the Case I and the Case II loading conditions.

    Except for symmetrical cable configurations, it will be necessary to determine vertical load eccentricity and post reactions in both transverse directions to find the maximum loads and stresses in the individual posts.

11. Investigate post adequacy for both Case I and Case II loading conditions;

    (a) Calculate the axial compressive stress (($f_a$ = P/A) at each post.

    (b) Determine the allowable compressive stress **($F_a$)** for each post.

    (c) Calculate the bending stress due to eccentricity ($f_b$) at each post using the formula:

    $$f_b = Pe/S$$

    where P is the post load calculated in step 10; e is the vertical load eccentricity, which is numerically equal to the horizontal displacement calculated in step 8; and S is the post section modulus.

(d) Use the combined stress expression to evaluate post adequacy. For this application, the expression is:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$$

In the expression, $f_a$ is the axial compressive stress, $f_b$ is the bending stress produced by vertical load eccentricity, and $F_a$ and $F_b$ are the allowable axial compressive and bending stresses, respectively.

For many cable-braced bents, stresses in the cap and sill beam may be determined by analysis in the usual manner; that is, using the Case II load combination. This procedure is usually satisfactory because the Case I load combination rarely governs cap or sill beam design. However, if the cables are attached near the end of a cap cantilever supporting a lightly loaded exterior beam, the Case I load combination, which includes the vertical component of the cable design load, may produce the maximum cap bending moment, and this should be kept in mind when reviewing cable-braced falsework bents.

4-5.07F(2) Completion of Analysis for Other Structure Types

The procedure described in the preceding section for box girder structures is generally applicable to slab and T-beam structures as well, except that for these structure types it is unnecessary to investigate system adequacy for an intermediate loading condition.

For the steps 10 and 11 calculations, the design load combination is the design live load, plus the design dead load, plus the vertical component of the cable unit design load, plus the horizontal design load.

For calculating post loads and stresses, all vertical design loads act on the falsework in its deflected position;

## 4-5.08 Cable Load Tests

In the absence of sufficient technical data to identify the cable and establish its safe working strength, the contractor may elect to perform one or more load tests. Judgment will be required as to the total number of tests needed.

For example, if the cable can be identified as to type and if it is in uniformly good condition, a single test may be sufficient for all cable of the same type. However, if the cable cannot be identified, or if it is old and obviously worn, it may be necessary to test each reel or drum furnished.

If a load test is needed to determine the physical properties of cable to be used in an internal bracing system, the test must be performed in a qualified testing lab. Field test results are not acceptable because determining cable properties such as the modulus of elasticity, the elastic stretch and the net metallic area of the cable requires precise measurements obtainable only with specialized testing equipment.

See also Section 4-6.02, Load Tests, for additional information pertaining to all types of load testing.

## 4-5.09 Cable Anchor Systems

In most cases cables will be secured by fastening the end to a concrete anchor block, although CIDH piles are sometimes used when relatively large forces must be resisted.

Concrete anchor blocks must be proportioned to resist both sliding and overturning. When checking anchor block stability, the weight of the anchor block must be reduced by the vertical component of the cable tension to obtain the net or effective weight to use in the anchorage computations.

For dry service conditions, the coefficient of friction assumed in the anchor block design should not exceed the following:

| | |
|---|---|
| Anchor block set on sand | 0.40 |
| Anchor block set on clay | 0.50 |
| Anchor block set on gravel | 0.60 |
| Anchor block set on pavement | 0.60 |

The tabulated friction values should be multiplied by 0.67 if it is likely the supporting material will become wet during the construction period.

For either concrete anchor blocks or CIDH piles, the method of connecting the cable to the anchorage is part of the design. The connecting device must be designed to resist both vertical (uplift) and horizontal forces.

If CIDH piles are to be used as cable anchors, the falsework drawings should show the pile diameter and length, the cement factor for concrete design, reinforcing steel details and cable anchor device. Additionally, since the load resisting capacity of a CIDH pile is dependant on the characteristics of the soil into which the pile has been cast, the contractor's design calculations should be given a cursory review to determine whether the assumed soil pressure and soil properties are consistent with the type of soil at the job site, and whether the design procedure follows recommended practice for piles subject to both uplift and lateral forces. Any inconsistencies should be brought to the contractor's attention immediately, and supplemental details and/or calculations requested. [19]

The adequacy of CIDH pile cable anchors should be evaluated in accordance with the methodology explained in Falsework Memo 9.

<u>4-5-10 Splicing Cable</u>

Because of the uncertainties associated with cable splicing, Division of Structures policy prohibits the use of splices in any cable used as falsework bracing.

[19] Design of piles to resist combined uplift and lateral forces is a sophisticated design procedure, which is sometimes approached superficially in the falsework design. However, the specification requirement for design calculations applies to piles and well as other elements of the falsework system. In the absence of calculations to support the design, the falsework submittal is not complete and the contractor should be so informed.

## Section 4-6 Manufactured Assemblies

### 4-6.01 Genera1 Information

In the specification context, the term "manufactured assembly" means any commercial product the use of which is governed by conditions and/or restrictions imposed by the manufacturer. Manufactured assemblies routinely used in falsework construction include products such as jacks, hangers, clips, brackets and similar hardware products as well as all types of manufactured shoring systems. When approved for use, such products may be incorporated into the falsework design, and upon construction of the falsework, they become a clearly identifiable component of the falsework system.

The specifications limit the load on, and/or the deflection of, any commercial product to the maximum recommended or allowed by the manufacturer of the product. Allowable loads and conditions or limitations of use must be shown in a catalog or technical. manual published by the manufacturer, or in a-statement of use compliance pertaining to a particular project.

A compliance statement, if furnished in lieu of catalog data, may be shown on a working drawing or included in a letter, but to be acceptable it must be signed by the manufacturer of the product under consideration, not by a material supplier or the contractor.

The specifications require the contractor to furnish technical data for manufactured assemblies, but only when such data is requested by the engineer. Therefore, the engineer should not hesitate to request catalog data or other technical information if it is needed to verify the load-carrying capacity of any manufactured product proposed for use in the falsework system.

Keep in mind that some manufacturer's catalogs are very brief and Show only general details. The engineer must be able to verify that the item is used as the manufacturer intended. Accordingly, if there is any question as to whether a device or product is being used properly, the contractor should be requested to furnish substantiating technical data or a statement of use compliance signed by the product manufacturer.

### 4-6.02 Load Tests

In any case where the falsework drawings show or describe a manufactured product or device which cannot be found in any catalog, the engineer should require a load test to establish the safe load-carrying capacity of that product or device.

The load test should be performed under conditions which will closely simulate the intended field use, particularly as to the method of support. If possible, the device should be tested to failure, in which case the safe working load may be assumed as one-half the ultimate load. This will provide a factor of safety of two, which is consistent with manufacturer's ratings for concrete form accessories. If it is not possible to test to failure, the working load used for the design should not exceed one-half of the maximum load carried during the test.

The procedure discussed herein for load testing of manufactured products or devices will also apply to noncommercial items, such as homemade deck overhang brackets, fabricated by the contractor. Note, however, that the required safety factor is increased to three for noncommercial products.

Load tests should be witnessed by the engineer and test results noted in the job records, since they form the basis of the determination of the allowable values to be used in the false-work design review.

When authorizing a load test, the engineer should keep the purpose of load testing in mind. Load testing is intended to determine or verify the load-carrying capacity of a commercially available product or device. Load testing should not be used to establish that a particular design detail or method of construction, although overstressed when stresses are calculated in the usual manner, is nevertheless capable of withstanding the imposed load without failing.

### 4-6.03 Beam Hangers

Beam hangers are hardware items which are placed transversely across the top flange of a beam or girder. Steel rods or bolts, which are inserted into threaded wire loops at the hanger ends, hang vertically to support the deck slab falsework or diagonally outward to support a deck overhang bracket.

Unbalancedloading (loading only one side of the hanger) will materially reduce the load-carrying capacity of the hanger unless it is of a type designed to be loaded on one side at a time, or unless special measures are taken to hold the hanger in place. Special measures may include welding to studs or shear connectors. In no case, however, should beam hangers be welded to the top flange of a steel girder or to precast-prestressed girder stirrups.

4-6.04 Deck Overhang Brackets

Several types of metal brackets specifically designed to support cantilevered deck overhangs are available commercially. Typically, these brackets will be supported at the top by beam hangers, or by form bolt inserts cast into concrete girder stems, or by threaded rods or bolts extending through holes drilled in the web of steel plate girders. Rotation of the bracket is prevented by a diagonal leg braced against the exterior girder.

The specifications governing steel construction include certain restrictions which affect the design of falsework supporting deck overhangs on steel girder bridges. (See Chapter 3.)

4-6.05 Steel Joist Assemblies

Joist assemblies are essentially steel beams which can be adjusted to provide a wide range of span lengths. Catalog data should be consulted to determine the safe load-carrying capacity. and the allowable deflection. Note that when joist assemblies are used to support deck slabs between girders, design load deflection is limited to the maximum deflection allowed by the manufacturer, which in some cases may exceed 1/240 of the span. At all other locations, the specification limit (l/240 of the span) applies.